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DISCUSSION OF TURBULENT TRANSFER MECHANISM AND SUSPENDED SEDIMENT IN CLOSED CHANNELS

(*Published in February, 1951*)

By Emment M. Laursen, M. R. Carstens,
and Hassan M. Ismail

HYDRAULIC DIVISION

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City Planning	58, 60, 62, 64, 93, 94. 99, 101, 104, 105, 115 (Discussion: D-16, D-23, D-43, D-60, D-62)
Construction	43, 50, 55, 71, 92, 94, 103, 108, 109, 113, 117 (Discussion: D-3, D-8, D-17, D-23, D-36, D-40)
Engineering Economics	46, 47, 62, 64, 65, 68, 69, 95, 100, 104 (Discussion: D-2, D-19, D-27, D-30, D-36, D-57)
Engineering Mechanics	41, 49, 51, 54, 56, 59, 61, 88, 89, 96, 116 (Discussion: D-5, D-XXIII, D-XXV, D-18, D-24, D-33, D-34 D-49, D-54, D-61)
Highway	43, 44, 48, 58, 70, 100, 105, 108, 113 (Discussion: D-XXVIII, D-7, D-13, D-16, D-23, D-60)
Hydraulics	50, 55, 56, 57, 70, 71, 78, 79, 80, 83, 86, 92, 96, 106, 107, 110, 111, 112, 113, 116 (Discussion: D-XXVII, D-9, D-11, D-19, D-28, D-29, D-56, D-70)
Irrigation	46, 47, 48, 55, 56, 57, 67, 70, 71, 87, 88, 90, 91, 96, 97, 98, 99, 102, 106, 109, 110, 111, 112, 114, 117, 118 (Discussion: D-XXIII, D-3, D-7, D-11, D-17, D-19, D-25-K, D-29, D-30, D-38, D-40, D-44, D-47, D-57, D 70)
Power	48, 55, 56, 69, 71, 88, 96, 103, 106, 109, 110, 117, 118 (Discussion: D-XXIII, D-2, D-3, D-7, D-11, D-17, D-19, D-25-K, D-30, D-38, D-40, D-44, D-70)
Sanitary Engineering	55, 56, 87, 91, 96, 106, 111, 118 (Discussion: D-10, D-29, D-37, D-56, D-60, D-70)
Soil Mechanics and Foundations	43, 44, 48, 94, 102, 103, 106, 108, 109, 115 (Discussion: D-4, D-XXVIII, D-7, D-43, D-44, D-56)
Structural	42, 49, 51, 53, 54, 59, 61, 66, 89, 100, 103, 109, 113, 116, 117 (Discussion: D-3, D-5, D-8, D-13, D-16, D-17, D-21, D-23, D-24, D-25-K, D-32, D-33, D-34, D-37, D-39, D-42, D-49, D-51, D-54, D-59, D-61)
Surveying and Mapping	50, 52, 55, 60, 63, 65, 68 (Discussion: D-60)
Waterways	41, 44, 45, 50, 56, 57, 70, 71, 96, 107, 112, 113, 115 (Discussion: D-8, D-9, D-19, D-27, D-28, D-56, D-70)

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DISCUSSION

EMMETT M. LAURSEN,[20] ASSOC. M. ASCE, and PIN-NAM LIN[21].—A fundamental issue unintentionally raised by this paper is the proper interpretation of experimental evidence. If the author's interpretation of his data is accepted, no fault can be found with his conclusions. However, a different interpretation can lead to conclusions that are just the reverse. Although the outcome of this discussion is an almost complete, but perfectly straightforward, disagreement with the author's views, the essence of the argument is far more subtle in that it hinges upon certain basic concepts of the philosophy of measurement.

Experimental research may be reduced schematically to three successive steps: (*a*) the direct measurement of a few easily perceived quantities; (*b*) the evaluation from these measurements, by means of definitions and physical laws, of other quantities not susceptible to direct observation; and, finally (*c*) the deduction of further laws from the correlated measurements and evaluations. It should be evident that the significance of the quantities measured or evaluated and of the numerical values assigned must be clearly understood if the laws that are deduced are to be statements of true physical relationships.

A few quantities, such as length and time, are usually susceptible to direct observation and, therefore, to direct measurement by comparison with known standards. The numerical values assigned by the measurement are a matter of convention. Questions may arise as to the precision of the measurement, but seldom does any misunderstanding arise as to what quantity has been given a numerical value or what the meaning of the value is.

Most so-called "measurements" are not this simple since many quantities are not susceptible to direct observation and hence cannot be measured directly by comparison with a known standard. The evaluation of such a quantity is then indirect. If a basic definition is used in the evaluation, no misunderstanding is likely to arise; however, if a physical law is used, the validity of the law is a pertinent question. Thus, if the velocity of an object is evaluated by timing it over a measured distance and using the definition $V = L/T$, no more questions arise than with direct measurement, even though the procedure is inherently different. On the other hand, if a pitot-tube assembly is used to obtain the velocity at a point in the flow, a very basic question arises. Is the quantity $V = C\sqrt{2\,g\,h}$ the same as the quantity $V = L/T$? The answer, of course, is in the affirmative provided that the Bernoulli equation is a true law, properly applied. Since this equation is well established, and since its limitations are well known, the actual questions are more likely to be on the technique level—that is, whether the coefficient C is applicable to the particular boundary and flow conditions.

This brief consideration of the concepts implicitly involved in so ordinary a derived measurement as that of velocity gives a clue to the difficulty that can

NOTE.—This paper by Hassan M. Ismail was published in February, 1951, as *Proceedings-Separate No. 56*. The numbering of footnotes, tables, equations, and illustrations in this Separate is a continuation of the consecutive numbering used in the original paper.

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arise in evaluations that depend on such incompletely established laws as those relating to sediment transportation. The numerical values of the coefficients λ, κ, and β on which the author bases his conclusions are such derived measurements based upon the application of certain approximate laws and procedures. In fact, the author's unquestioning acceptance and use of the Kármán-Prandtl velocity equation and its counterpart for sediment distribution has led to conclusions that, though they seem to have fundamental significance, are in no way justified by a more careful analysis. The writers, therefore, submit herewith a detailed criticism of the author's principal points, together with an independent analysis in which each indirect evaluation is carefully examined.

Since λ is defined by the Darcy-Weisbach formula, no other relationship can be used for the computation of its numerical values. Furthermore, any other coefficient of resistance, such as the Manning n, depends on exactly the same values of flow (Q), area (A), hydraulic radius (R), friction head (h_f), and length (L) and, therefore, should exhibit a similar pattern of variation. The significance of the quantity λ is that it is a coefficient that is required to make the Darcy-Weisbach formula an equality. The accuracy of its evaluation, however, depends on the accuracy of the measurements of the more basic quantities just listed. With regard to the measurement of these quantities, only two questions can be raised: one in connection with h_f, which is usually difficult to measure precisely; and one in connection with A and R, as their effective values may be influenced appreciably by the depth of sand in the flume. However, since it is reasonable to suppose that the dunes would act as roughness elements, the results are in qualitative agreement with past experience. Therefore, the author's conclusion in the section on "Discussion of Results" that the dunes and not the suspended sediment affect the resistance to flow would seem to be substantiated, despite the scatter of the experimental points.

In contrast, perhaps the most striking example of an unwarranted conclusion is the one—based on Fig. 9, in which κ is plotted against the total sand "load" —that κ decreases with increasing concentration and that the decrease indicates damping of the turbulence by the suspended sediment. The total sand "load," which could be more expressibely termed the total sand "charge," actually bears little relation to the concentration except as an upper limit. In the examination of this conclusion, κ has been plotted in Fig. 16 against the concentration at $h = (y_m/y) - 1 = 20$, as representative of the concentration in the section. Although the κ values are generally lower for higher concentrations, the scatter is too great to allow any convincing conclusion to be drawn as to cause and effect. Therefore, it is not only proper, but advisable, to inquire into the significance of the κ values.

The computation procedure for the determination of the κ values is the use of the Kármán-Prandtl velocity equation, according to which κ is the ratio of the shear velocity to the slope of the line through the u: log y values. If κ is also to be considered as a measure of the turbulence, the numerical values are the result of the semi-logarithmic equation and the numerical values of u, y, and U_f. (It might be pointed out that von Kármán's original

kappa (κ_o), in which $l/\kappa_o = \frac{du/dy}{d^2u/dy^2}$ or $\sqrt{\tau_o/\rho} = -\kappa_o \frac{(du/dy)^2}{d^2u/dy^2}$ is not, as implied, the same as the kappa (κ) the author has computed. They are related by $\kappa_o = \sqrt{1 - y/y_m}\,\kappa$.) Since the variation in κ could not be explained satisfactorily in this manner, the computation of the κ values was checked by replotting the velocity profiles and by distributing the shear according to the method of H. A. Einstein,[11] Assoc. M. ASCE, but without sensibly different results. The lack of correlation may be the result of one or more unknown reasons—most plausibly a lack of precision in the original direct measurements or an inability of κ to express fully the turbulence properties of the flow. Therefore, it was concluded that another procedure involving fewer derived values was needed for the evaluation of the effect of the suspended sediment on this characteristic of the flow.

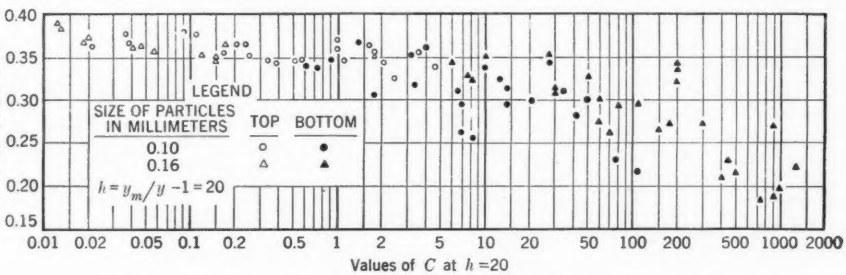

FIG. 16.—VALUES OF κ VERSUS CONCENTRATION AT $h = (y_m/y) - 1 = 20$

If the turbulence pattern is modified by the presence of sediment, such an effect should be detectible in the shape of the velocity profile. If one assumes that the velocity distribution may be written in the exponential form:

$$u = a\, y^{1/m} \quad \text{.........................(15)}$$

m will describe the shape of the profile. This form has the advantage that m is evaluated from the u: y values alone and not from any relationship for the shear. The excellence of the fit of the experimental points in Fig. 17 indicates the reasonableness of the assumption. For the more pertinent runs, m values are given in Table 2 together with the total charge, the mean concentration, the estimated inert charge, and the author's κ values. A definite system can be observed in the variation of m. For any total charge, the m value is smallest for the lowest concentration and greatest for the highest concentration. Since the concentration and the inert charge are inversely related, for any total charge the m value is smallest for the largest inert charge and increases as the inert charge decreases. In fact, if the velocity is sufficient to sweep most of the sediment into suspension, m approaches the value for clear water. From this it can be inferred that the suspended material has little, if any, effect on the velocity distribution. Since the head loss is also unaffected by the concen-

[11] "Formulas for the Transportation of Bed Load," by H. A. Einstein, *Transactions*, ASCE, Vol. 107, 1942, p. 575.

tration, neither the turbulence production nor the turbulence distribution can be influenced by the presence of the suspended sediment. Instead, the major effect seems to be caused by the material on the bed, either because of roughness or because of transport as bed load. Unfortunately, the inert charge is not a sufficient criterion to permit quantitative description of this effect. However, the author's observations (Table 1, Col. 23) tend to substantiate the writers' conclusion that bed conditions alone govern the variation of m.

It is interesting to note that J. Nikuradse[22] found a similar variation of m with roughness in his classic experiments, the results of which are summarized in Fig. 18. For his roughest condition ($r_O/k = 15$) m was equal to 4.2. A comparable equivalent roughness height for the author's flume would be 0.1 in., which could easily be produced by the inert charge in the shape of dunes. In fact, the author cites dunes 0.05 in. high in Run 120 and a sand wave 0.6 in. high in Run 119.

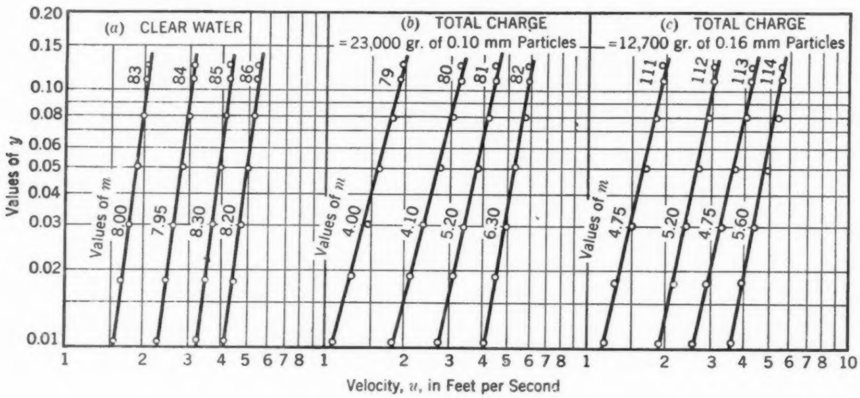

Fig. 17.—Comparison Between Exponential and Measured Velocity Distributions

All the questions raised in regard to velocity distribution may be repeated in regard to sediment distribution for two reasons: (1) basically, since both depend on turbulence; and (2) particularly, since the author's determination of sediment distribution implicitly involves the Kármán-Prandtl velocity equation. Specifically, β as determined by the author equals $\frac{w}{\kappa\, U_f}$ divided by the slope, z, of the line representing log C versus log h, and $\frac{\epsilon_{sc}}{\beta\, \kappa\, y_m\, U_f}$ equals a constant 0.22. The Kármán-Prandtl equation is implicitly used twice in the computation of β—once in the evaluation of κ and then again in the expression of the form of the sediment distribution equation. In the instance of ϵ_{sc} it is only involved in the form of the sediment-distribution equation, since $z = w/\beta\, \kappa\, U_f$. It is thus clearly apparent that sediment distribution cannot be considered independently of velocity distribution. Moreover, this examina-

[22] "Stromungsgestze in Rauhen Rohren," by J. Nikuradse, VDI Forschungsheft 361, 1933.

tion reveals that Fig. 14 is obtained from the averaged reverse order of the above computational procedure. Therefore, it is not indicative of the validity of the assumptions or conclusions, but merely of the correctness of the computations—the derived β values necessarily forcing the theoretical curve to fit the data with good approximation.

TABLE 2.—VALUES OF m FOR SELECTED RUNS

Run (1)	Total sand charge, W (2)	Concentration (3)	Inert sand charge (4)	m (bottom) (5)	Universal constant, κ (bottom) (6)
65	3,600	0.92	3,065	5.65	0.313
66	3,600	2.90	1,910	6.60	0.293
67	3,600	3.08	1,810	7.25	0.299
68	3,600	3.10	1,800	7.25	0.346
69	6,800	1.49	5,935	4.95	0.300
70	6,800	5.40	3,660	5.30	0.230
71	5,800	4.77	3,450	6.85	0.283
72	6,800	6.68	2,910	7.35	0.310
73	13,200	2.39	11,810	4.40	0.260
74	13,200	7.23	9,000	4.30	0.195
75	13,200	12.57	5,890	4.70	0.216
76	13,200	12.65	5,840	5.30	0.216
77	13,200	12.28	6,050	7.15	0.256
78	13,200	12.38	6,000	8.25	0.294
79	23,200	2.92	21,500	4.00	0.285
80	23,200	13.66	15,250	4.10	0.201
81	23,200	31.10	5,100	5.20	0.221
82	23,200	23.00	9,850	6.30	0.268
83				8.00	
84		CLEARWATER		7.95	
85				8.30	
86				8.20	
103	3,100	0.38	2,875	6.60	0.351
104	3,100	1.94	1,970	6.20	0.311
105	3,100	2.21	1,815	6.50	0.300
106	3,100	2.16	1,845	7.65	0.327
107	6,300	0.45	6,040	6.25	0.353
108	6,300	3.10	4,490	5.75	0.292
109	6,300	5.00	3,400	5.75	0.265
110	6,300	4.70	3,570	6.40	0.295
111	12,700	0.57	12,370	4.75	0.262
112	12,700	5.10	9,740	5.20	0.274
113	12,700	10.70	6,480	4.75	0.210
114	12,700	13.80	4,670	5.60	0.230
115	25,500	1.3	24,740	3.30	0.272
116	25,500	4.8	22,710	4.30	0.272
117	25,500	13.5	17,680	4.00	0.215
118	25,500	24.0	11,550	4.55	0.199
119	38,300	1.4	37,490	3.85	0.272
120	38,300	4.5	35,680	3.65	0.221
121	38,300	13.0	30,750	3.45	0.187
122	38,300	30.0	20,800	3.90	0.183

Another factor neglected by the author that should be considered in any computational procedure to determine ϵ_s or β is the variation of the fall velocity, w, with the concentration. Thus, Eq. 1 should read:

$$\epsilon_s = \beta\ \epsilon_m = \frac{w_{ac}\ C}{-dC/dy} \qquad (16)$$

in which w_{ac} is the actual fall velocity. The actual fall velocity for a homogeneous suspension of uniform particles is given[23] in Fig. 19 as a function of the concentration and of the Reynolds number. This correction factor can be sizable (for a 0.1-mm sand at a concentration of 30 grams per liter the decrease in fall velocity is about 20%) and should not be neglected. Although an analytic expression can be written for the correction factor, its inclusion makes the integration of Eq. 16 too formidable (the expression for w_{ac}/w contains fifteen terms). Nevertheless, an approximate evaluation indicates that use of the actual fall velocity would decrease the author's β values, but not sufficiently to change his conclusion qualitatively—that is, β would still be greater than unity, whereas the following considerations require that it be equal to or less than unity.

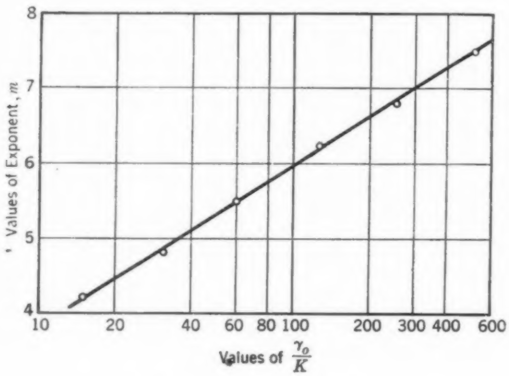

FIG. 18.—VARIATION OF m WITH ROUGHNESS (AFTER NIKURADSE)

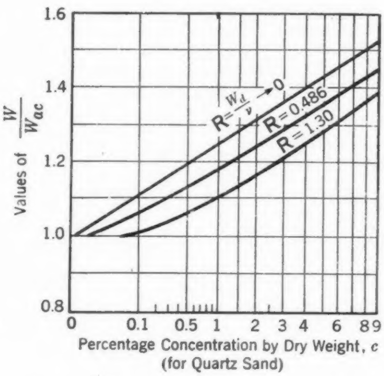

FIG. 19.—EFFECT OF CONCENTRATION ON FALL VELOCITY

Since the questions that arose in regard to κ and U_f in the velocity distribution analysis still remain, the writers have chosen to integrate the differential equation for sediment distribution on the basis of an exponential velocity distribution. Thus, the momentum-exchange coefficient becomes

$$\epsilon_m = \frac{\tau/\rho}{du/dy} = \frac{\tau_o/\rho\ (1 - y/y_m)\ y}{U_{\max}/m\ (y/y_m)^{1/m}} \qquad (17)$$

Inserting Eq. 17 into Eq. 16, rearranging terms, and intergrating produces:

$$\ln \frac{C}{C_a} = \frac{w_{ac}\ (1 + 1/m)^2}{\beta\ U_{\max}\ \lambda}\ [I_\eta - I_\alpha] \qquad (18)$$

in which

$$I_\eta = \frac{1}{m} \int_o^\eta \frac{d\eta}{\eta^{\frac{m-1}{m}}\ (\eta - 1)}$$

[23] "Effect of Spacing and Size Distribution on the Fall Velocity of Sediment," by Pin-Nam Lin, thesis presented to the University of Iowa, at Iowa City, Iowa, in 1951, in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

and

$$\eta = y/y_m, \; \alpha = a/y_m.$$

The necessary values of the I-function have been obtained numerically and are presented in Fig. 20. (By chance, an equivalent function has been tabulated by B. A. Bakhmeteff as a varied-flow function.[24] His values have therefore been incorporated into the figure.)

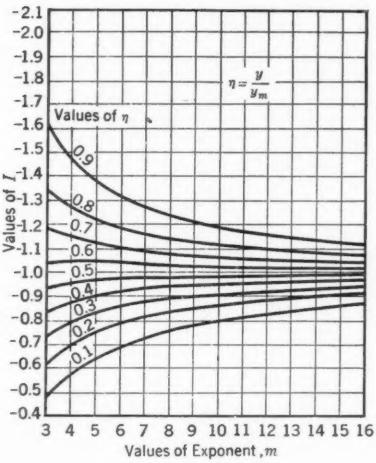

FIG. 20.—VALUES OF I-FUNCTION FOR SEDIMENT DISTRIBUTION

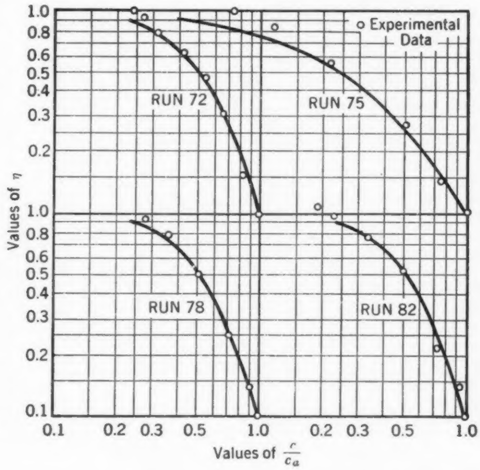

FIG. 21.—COMPARISON BETWEEN EQ. 18 AND MEASURED SEDIMENT DISTRIBUTION

Fall velocities corrected for the concentration at mid-depth and λ values from Eq. 12 were used in the theoretical curves of Fig. 21. The curves of Fig. 21 are based on the information given in Table 3 used in Eq. 18 with a value of $\beta = 1$.

TABLE 3.—DATA FOR CURVES OF FIG. 21

Run	Actual fall velocity in terms of theoretical (w_{ac}/w)	Percent concentration $C\ y_{m/2}$	Friction coefficient λ	Maximum velocity U_{max}	m
72	0.85	1.35	0.0157	5.65	7.60
75	0.81	2.80	0.0170	3.72	4.67
78	0.81	2.85	0.0152	5.86	7.25
82	0.76	5.20	0.0150	6.07	6.85

Although in all runs the β values are very close to unity, the theoretical treatment has been approximate, and another consideration indicates that the true β value, as defined by the ratio of the mixing coefficients of sediment and momentum, would be less than unity. There is, of course, a sorting of

[24] "Hydraulics of Open Channels," by B. A. Bakhmeteff, McGraw-Hill Book Co., Inc., New York, N. Y., 1932, p. 310.

the sediment in the vertical such that the mean size of the particles decreases with increasing y values. For this reason, and possibly others, the mean size of the material in suspension is smaller than the mean size of the material on the bed. Since finer material would have a smaller fall velocity, the derived β values would be less than unity if the data included this factor. The sorting effect would also explain the deviation of the theoretical curve from the experimental points as the ratio y/y_m approaches 1.

An independent analysis of β has been made, both theoretically and experimentally, by M. R. Carstens,[25] Jun. M. ASCE, for a quartz sphere in simple harmonic motion. As a first approximation, Fig. 22 can be used to evaluate β as a function of the frequency ω, the amplitude of the motion having a negligible role. Although a weighted mean of the turbulence frequencies would have to be employed, it is evident that, according to the analysis by Mr.

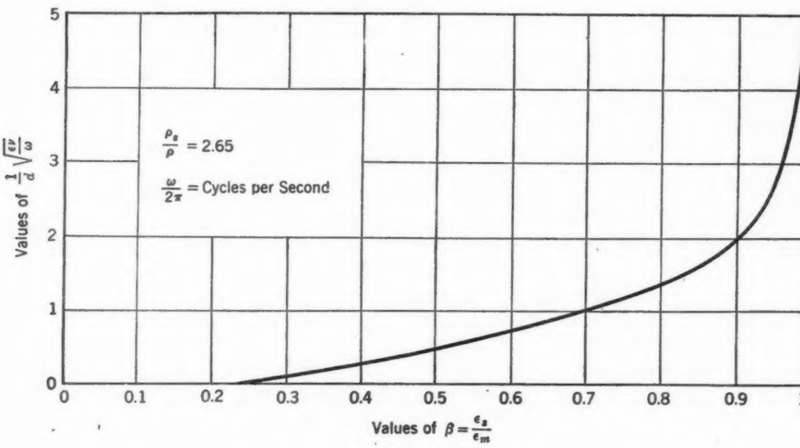

FIG. 22.—VARIATION OF β WITH FREQUENCY AND PARTICLE DIAMETER (AFTER CARSTENS)

Carstens, β should always be less than unity. The qualitative conclusion that ϵ_s is smaller than ϵ_m is thus checked independently. In order to obtain a quantitative relation between ϵ_s and ϵ_m by either method, further data would be necessary. In the one method the turbulence frequencies, and in the other the size frequency of the suspended sediment, must be known.

The exponential velocity distribution has been used herein as a matter of convenience and not as an argument against the logarithmic form. By this expedient the possible errors in the head-loss measurements, which are reflected in the shear values, have been minimized. The point in question is not which should be used, for both are assuredly empirical and approximate. Admittedly, Eq. 18 is also merely an approximation even without considering the variation of β with y and of w with c. However, even if a completely satisfactory analysis can not be made, the rigorous examination of each procedural step has indicated

[25] "Accelerated Motion of a Sphere," by M. R. Carstens, Doctoral Dissertation, State University of Iowa, Iowa City, Iowa, 1950.

that the experimental data do not substantiate the author's conclusions except in regard to λ. In fact, the writers' analysis leads to just the opposite conclusions:

1. That the nature of the quantity κ is not sufficiently well defined to allow reliable conclusions to be drawn from the variation of its values;
2. That the presence of suspended sediment has little or no effect on the flow; and
3. That β is equal to or less than unity.

M. R. CARSTENS,[26] JUN. ASCE.—The general diffusion process is characterized by the differential equation[27]

$$N_R = -D_R\, dn/dy \qquad (19)$$

in which N_R is the net rate of transfer of the fluid characteristic, D_R is the diffusion coefficient, and n is the characteristic concentration. Since the net rate of transfer is in the direction of decreasing concentration, the sign is negative. This process is represented by the suspended sediment diffusion process in the equilibrium state, in which the net rate of transfer of sediment upward is equal to the rate of settling, that is:

$$w\, c = -\epsilon_s\, dc/dy \qquad (20)$$

The magnitude of the diffusion coefficient ϵ_s is representative of the intensity of the transfer mechanism in the y direction. The diffusion equation of fluid momentum is identical in form. The diffusion coefficient ϵ_m is representative of the fluid transfer mechanism in the y direction. Since momentum is a characteristic of the fluid, the diffusion coefficient of momentum is in reality a diffusion coefficient of the fluid. Since the suspended sand is of greater density than the water, the sand particles will be out of phase with the movement of the fluid particles and will be transported a lesser distance than the fluid particles. Consequently, the momentum diffusion coefficient ϵ_m is never smaller in magnitude than the sediment diffusion coefficient ϵ_s if the sediment is greater in density than the fluid.

Studies by the writer[28] and by M. Wagenschein[29] were performed with a spherical particle in order to determine the ratio of the particle amplitude to the fluid amplitude. There is available for the amplitude ratio an analytical solution in which the ratio is less than 1.00 if the foreign particle is more dense than the fluid particle. Conversely, if the fluid particle is less dense, then the amplitude ratio is between the limits of 1.00 and 3.00. In other words, in Eq. 2, $O < \beta < 1.00$ if $\rho_s/\rho > 1.00$ and $1.00 < \beta < 9.00$ if $\rho_s/\rho < 1.00$. The

[26] Associate Prof., School of Civ. Eng., Georgia Inst. of Technology, Atlanta, Ga.

[27] "Engineering Hydraulics," edited by Hunter Rouse, John Wiley & Sons, Inc., New York, N. Y., 1st Ed., 1950, p. 95.

[28] "Accelerated Motion of a Sphere," by M. R. Carstens, thesis presented to the University of Iowa, at Iowa City, Iowa, in 1950, in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

[29] "Experimentalle Untersuchung über das Mitschwingen einer Kugel in einer Schwingenden Flüssigkeit-oder Gasmasse," by Martin Wagenschein, *Annalen der Physik*, Series 4, Vol. 65, 1921, pp. 461–480.

upper limit corresponds to the case of air bubles in water, for which ρ_s/ρ is approximately zero.

The Boussinesq relationship for the apparent shear stress in turbulent flow is

$$\bar{\tau} = \rho\, \epsilon\, \overline{du'}/dy \qquad (21)$$

in which the bars denote temporal mean values. A rational derivation for the apparent shear stress can be performed by means of a momentum flux equation applied to a small fluid element. From this derivation the apparent shear stress is

$$\bar{\tau} = -\rho\, \bar{v}'\, \bar{u}' \qquad (22)$$

in which u' and v' are the fluctuating components of the velocity in the x and y directions, respectively. Measurements[30] of the velocity fluctuations in a two-dimensional channel verify the validity of Eq. 22. Comparing the terms of Eqs. 21 and 22, it is apparent that the fluctuating component v' is the lateral transport velocity and, hence, is proportional to the diffusion coefficient ϵ. It then follows that $\overline{du}/dy$ in Eq. 21 is proportional to u'. Since the diffusion coefficient or eddy viscosity ϵ contains this proportionality factor, the magnitude of ϵ is not equal to the diffusion coefficient of fluid or momentum. In fact, Eq. 21 is useful mainly as an analogy to the viscous shear of laminar flow and is a descriptive aid in explaining the apparent shear of turbulent flow. C. B. Millikan[31] has shown that the logarithmic velocity distribution in channels and pipes can be derived without the use of Eq. 21 and with the aid of more fundamental assumptions.

The general diffusion equation (Eq. 19) will result in a zero net rate of transfer when either the diffusion coefficient D_R is zero or the gradient of the quantity being transported, dn/dy, is zero. As the center line of a two-dimensional channel the net rate of transfer of momentum is zero because the gradient of momentum is zero. The diffusion coefficient of momentum ϵ_m is not zero at the center line of the channel. As a consequence, fluid, sediment, coloring matter, heat, and the like are transported across the center line when there is a gradient of these quantities.

HASSAN M. ISMAIL,[32] ASSOC. M. ASCE.—The final results of any research definitely depend on the interpretation of experimental evidence. The selection of the functions and the form in which they appear in the different relations are among the most important parts of the research. A comparison between Figs. 9 and 16 will illustrate the effect of grouping the experimental data in a manner that will make them representative. In Fig. 16 the concentration at a certain value $h \left(= \frac{y_m}{y} - 1\right) = 20$ was taken as a base for studying the variation of the values of K. The scatter was too great. When the experimental points were grouped and plotted as in Fig. 9, a definite relationship

[30] "Investigation of Turbulent Fow in a Two-Dimensional Channel," by John Laufer, *Technical Note No. 2123*, National Advisory Committee for Aeronautics, 1950.

[31] "Turbulent Flows in Channels and Circular Tubes," by C. B. Millikan, *Fifth International Congress of Applied Mechanics*, John Wiley & Sons, Inc., New York, N. Y., 1939.

[32] Civ. Engr., Ministry of Public Works, Cairo, Egypt.

was shown. Fig. 7 shows that, for the same total quantity of sand, the curve of concentration for different velocities cross each other at different values of y. This is why the concentration at a certain depth could not be taken as a basis for comparison. Other factors discouraged the use of a big value of h, such as $h = 20$ (that is, $y = 0.006$ ft). The concentration curve at the bottom was very steep, and any small error in evaluating y would cause an appreciable error in the value of C_a. Also, using a value of h anywhere in the bottom half would not permit using results of the upper half. For these reasons the curves in the paper did not involve a reference plane, except Fig. 14 for which the reference plane was taken at the center of the profile.

It seems that τ, the shear stress, must have been confused with τ_0, the shear stress at the boundary, in the discussions on κ and κ_0 by Messrs. Laursen and Lin. The Prandtl equation is

$$\tau = \rho l^2 \left(\frac{du}{dy}\right)^2 \qquad (23)$$

Also:

$$\frac{l}{\kappa_0} = \frac{du/dy}{d^2u/dy^2} \qquad (24)$$

Therefore:

$$\sqrt{\frac{\tau}{\rho}} = \kappa_0 \frac{(du/dy)^2}{d^2u/dy^2} \qquad (25)$$

Eq. 25 contains τ and not τ_0. The relation between τ and τ_0 is expressed as:

$$\tau = \tau_0 \left(1 - \frac{y}{y_m}\right) \qquad (26)$$

and κ_0 in Eq. 25 is the same as κ used in the paper.

Eq. 15 may be a very good approximation expression for the velocity distribution; but since the velocity distribution is mainly dependent on the shear stress, m must be a function of the shear stress. The advantage of the von Kármán formula is that the shear stress appears in the formula, leaving K as a universal constant that depends only on the mechanism of the turbulence. The advantage of being able to evaluate m from the u-values and y-values alone is the same as that of introducing a new factor—say, $n = \frac{K}{u_f}$— in the velocity defect law, $u - u_{max} = \frac{2.3}{n} \log \frac{y}{y_m}$. As a matter of fact, n will be a combined function, and there is no doubt that the use of $\frac{K}{u_f}$ is very advantageous.

The conclusion of Messrs. Laursen and Lin that the presence of suspended sediment has little or no effect on the flow, was based on values of that combined function m. They state that, as the velocity increases, m approaches the value for clear water. The results given in Table 2 for runs 103 to 122 do not agree with this conclusion.

Mr. Carstens[28] as well as Messrs. Laursen and Lin[23] base their conclusion, that β must be less than unity, on research at the University of Iowa, Iowa City, Iowa. The results reported by T. K. Sherwood and B. B. Woertz[4] and by W. Corcoran,[19] as well as the studies reported in the paper, prove that β can be greater than unity.

[28] "Accelerated Motion of a Sphere," by M. R. Carstens, thesis presented to the University of Iowa, at Iowa City, Iowa, in 1950, in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

[23] "Effect of Spacing and Size Distribution on the Fall Velocity of Sediment," by Pin-Nam Lin, thesis presented to the University of Iowa, at Iowa City, Iowa, in 1951, in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

[4] "Mass Transfer Between Phases," by T. K. Sherwood and B. B. Woertz, *Industrial & Engineering Chemistry*, Vol. 31, 1939, p. 1034.

[19] "Temperature Gradients in Turbulent Air Streams," by W. Corcoran, thesis presented to the California Inst. of Technology at Pasadena, Calif., in 1948, in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

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JANUARY, 1952

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

DISCUSSION OF DESIGN OF IRRIGATION SYSTEMS

(*Published in February, 1951*)

By Ahmed Shukry, and W. H. Nalder

IRRIGATION DIVISION

Headquarters of the Society
33 W. 39th St.
New York 18, N.Y.

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DISCUSSION

Ahmed Shukry,[2] Assoc. M. ASCE.—In the limited extent of the paper, the author has covered intelligently a subject of indefinite limits. However, the expression (under the heading "Influence of Economics on Design") "full utilization of the water allocated to irrigation" needs some broad explanation, especially when related to the future of irrigation in general. The estimated field water duties for the crops on the irrigated land directly affect the efficiency, cost, and design of any irrigation project. The capacities of storage reservoirs, the sizes of diversion and headworks, conveyance structures, regulating structures, protection structures, and drainage systems all depend primarily on these duties. The future development of profitable irrigation schemes will depend largely on the extensive scientific and experimental studies of the relations between the yields of the different crops and water supplied. At least, this may be attempted in densely populated countries that have large marginal cultivable areas but possess limited sources of irrigation water. The latter case may be represented by Egypt.

In Egypt the area cultivated in 1950 was nearly 6 million feddans (1 feddan = 1.038 acres) and the population was over 19 million. The country is mainly dependent on agriculture, the agricultural products representing approximately 90% of the national income. The population is rapidly increasing, and since 1900 it has nearly doubled. The additional area that can be cultivated was estimated in 1946[3] as 2.0 million feddans. However, studies of the nature of the soil in the deserts east and west of the Nile Delta show that the total cultivable area may reach 10 million feddans, especially when power becomes cheap and the low regions of these deserts are irrigated by pumping. Because of the rapidly increasing population, new land must be continuously reclaimed for agriculture, to avoid the possible starvation that would otherwise ensue in a few years. The area of land under cultivation per capita (in feddans) has fallen from 0.48 in 1907 to less than 0.30 in 1951.

The main water source in Egypt is the River Nile. The water required for Egypt and the Sudan is stored during the high stage of the river in three reservoirs, namely, Aswan, Sennar, and Gabel Awlia (Fig. 4). These reservoirs are in the river channel itself. The majority of the flood water, being heavily charged with silt, is wasted into the sea.

To enable the future extension of cultivation, a huge storage scheme for utilizing all the Nile sources is under discussion,[4,5] and its two major provisions can be summarized as follows:

Note.—This paper by W. H. Nalder was published in February, 1951, as *Proceedings-Separate No. 57*. The numbering of footnotes and illustrations in this Separate is a continuation of the consecutive numbering used in the original paper.

[2] Asst. Prof. of Irrigation, Faculty of Eng., Farouk First Univ., Alexandria, Egypt.

[3] "The Future Conservation of the Nile," by H. E. Hurst, R. P. Black, and Y. M. Simaika, *Paper No. 51*, Ministry of Public Works, Govt. Press, Cairo, Egypt, 1946.

[4] "Report of the Consultant Committee on Storage Projects," Ministry of Public Works, Govt. Press, Cairo, Egypt, 1948.

[5] "Future Control of the Nile Sources," by Ali Bey Fathy, Faculty of Engineering, Farouk First Univ., Alexandria, Egypt, 1949.

1. Annual storage, by which the water requirements for a single year will be stored in the river channel itself, in the existing reservoirs, and in a fourth future reservoir at Merowe.

2. Over-year storage, by which the water will be stored in the watershed areas of the river, namely, Lake Victoria, Lake Albert, and Tana Basin. This water will be used to fill the annual reservoirs in low flood-years.

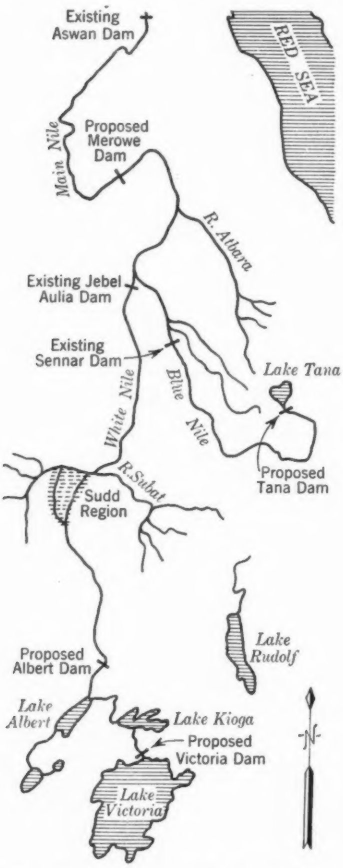

FIG. 4.—EXISTING AND PROPOSED DAMS IN THE UPPER NILE BASIN IN 1951

In spite of this huge program for controlling all the Nile sources and for reducing the losses in the Sudd region, the stored water will be just sufficient to irrigate 7.1 million feddans in Egypt and 2.0 million feddans in the dry regions of Sudan.

Different proposals are being studied for providing irrigation water to the remaining 2.9 million feddans in Egypt or to any other land that may be reclaimed in the distant future. The summary of four of these proposals is as follows:

a. Drawing the irrigation water by pumping from wells extending through the pervious substratum;

b. Re-use of the irrigation water by irrigating from drains;

c. Reducing the available conveyance losses of irrigation water; and

d. Economy in the use of water for the irrigated lands.

A final opinion on these proposals has not yet been given. However, it is believed that the economical use of irrigation water, based on long scientific and experimental studies, might be the last frontier against the dwindling ratio of cultivated area to population.

The economical use of irrigation water can be explained briefly as follows: The published results of the experiments carried out on the various factors affecting the relation between the crop yield and irrigation water[6,7] show, for the majority of crops, a general trend as represented schematically in Fig. 5. Under constant conditions of the other affecting parameters, the yield increases

[6] "Irrigation Practice and Engineering," by B. A. Etcheverry and S. T. Harding, McGraw-Hill Book Co., Inc., New York, N. Y., Vol. I, 1933.

[7] "Irrigation Principles and Practices," by Orson W. Israelsen, John Wiley & Sons, Inc., New York, N. Y., 1950.

in an approximately linear relationship with the amount of irrigation water up to a certain point (A). From point A to the point of maximum yield B the curve has a diminishing slope, and near point B it is generally so flat that

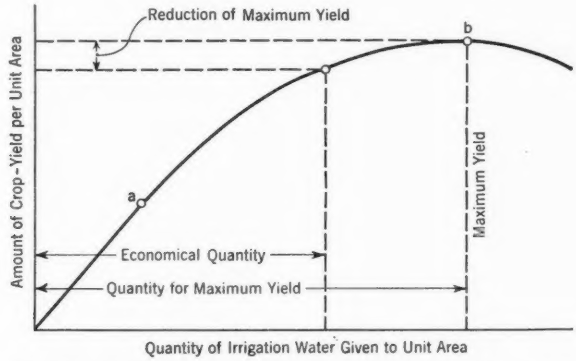

FIG. 5.—GENERAL RELATIONSHIP BETWEEN CROP YIELD AND THE QUANTITY OF IRRIGATION WATER

big amounts of water can be saved with little reduction of the maximum yield of the crop.

True knowledge of such relations may, therefore, produce economies in the irrigation water that may be given to some crops beyond their points of maximum yield. Furthermore, the available irrigation water in a district may not be sufficient to produce the maximum yields of the crops in the whole cultivable area. In such districts it may be more economical to use water duties less than those that produce the maximum yields and cultivate the whole area, rather than cultivating part of the area under the condition of maximum yield.

W. H. NALDER,[8] M. ASCE.—The discussion is a valuable contribution to this paper. The points raised by Mr. Shukry on the need for comprehensive planning of irrigation systems are cogent and deserving of the emphasis that he has placed on them. His suggestion that the paper should have devoted additional coverage to the problems of "full utilization of the water allocated to irrigation" is well taken. However, because of the limited space, this aspect of the paper was necessarily abridged.

Mr. Shukry's summary account of the proposed extension of irrigation developments in Egypt are of considerable interest and will no doubt receive widespread attention by all members of the profession who are engaged in furthering their countries' water resources and conservation developments.

[8] Chf. Designing Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

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